843

VOLUME 81 PAPER No. 843

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

DECEMBER, 1955

DISCUSSION OF PROCEEDINGS PAPERS

475, 477, 513, 550, 570, 754,
755, 756

SOIL MECHANICS AND FOUNDATIONS
DIVISION

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Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

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This paper was published at 2500 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N.Y.

Discussion of
"SAFE LOADS ON DOG-LEG PILES"

by James D. Parsons and Stanley D. Wilson

JAMES D. PARSONS,[1] M. ASCE, and STANLEY D. WILSON,[2] A.M. ASCE.—The allowable loads permitted on the piles described by the authors were limited by design stresses stipulated in the New York City Building Code. The fact that test piles successfully withstood loads in excess of twice the design loads does not indicate that the design was unduly conservative with the limiting stresses permitted by the Code.

Mr. Cummings makes a strong plea for the acceptance of load tests on dog-leg piles in lieu of the analysis presented in the paper. While the authors also urge the performance of load tests, they disagree with Mr. Cummings' conclusion that load tests alone should be used in the final analysis. If the tip of the pile cannot be seen with a lowered light, or if the pile cannot be dewatered, then the field engineer has no way of knowing, unless measurements are taken, how much the pile is bent. Pile-load tests, if performed on every pile suspected of being bent, would cost more than the replacement value of the piles.

The results of full-scale field tests such as those referred to by Mr. Cummings are quite interesting but unfortunately do not tell the entire story. All the tests conducted to date with which the authors are familiar have been short-time tests, and there is no question but that most of the load is transferred to the soil from the upper portion of the pile and only a small percentage reaches the tip. If this condition remained unchanged with time, it would mean that a lot of money has been wasted on over-length piles. In reality, of course, the compressible soils in the upper region consolidate and settle slowly with time, transferring more and more of the load to the tip of the pile. This is particularly true of piles penetrating through sands overlying a compressible stratum where load tests would not reflect the true performance of the piles over an extended period. It is, therefore, important that the measurement of the curvature of the piles and an analysis of the stresses be carried out in all instances where the piles are known to be badly bent, irrespective of the results of load tests.

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Discussion of
"FOUNDATION CONDITIONS IN THE
CUYAHOGA RIVER VALLEY"

by Ralph B. Peck
(Proc. Paper 513)

RALPH B. PECK,[a] M. ASCE.—Professor Casagrande has suggested that the low values of skin friction on piles in the overconsolidated clays are at least partly a consequence of the observed high excess hydrostatic pressures. These pressures would prevent reconsolidation of the clay after pile driving. The writer agrees that this factor may play an important part in the observed behavior of the piles.

Both Mr. Baker and Professor Casagrande have described an interesting project in which the compressible alluvial deposits were successfully consolidated by preloading the site under a unit load some 50 per cent in excess of the proposed dead and live loads. This technique deserves consideration on future projects in the valley when consolidation of the underlying clays will not be excessive. A careful distinction should be made between soft alluvial deposits such as those represented by Area A, Fig. 4, and more cohesive and less pervious deposits such as those represented by Area D. The procedure might be unnecessary or less effective in connection with deposits of the latter type.

Professor Casagrande has suggested the addition of information on the settlements of heavy structures on friction piles in the clay. Unfortunately, there appear to be very few data of this kind where deep-seated settlement or negative skin friction are not complicating factors. The following examples summarize such information known to the writer.

A steam power plant, located in Area C (Figs. 3 and 5) was constructed in 1943 on Union Metal piles from 70 to 80 ft long, loaded to 30 tons each. The settlements of 12 columns in the boiler and turbine rooms were measured in 1955. They varied from 0.003 ft to 0.041 ft.

A retaining wall for an ore-retaining structure, similar in position to the wall at a distance of 62 ft from the river in Fig. 11 but located between Areas B and C (Fig. 3) rests on 40-ft timber piles extending through about 15 ft of alluvium and 25 ft of clay. The settlement over a period of about 30 years has been as much as 7 inches, but the movements are undoubtedly associated to some extent with deep-seated displacements due to the weight of the ore in storage.

A recent publication containing contours of the bedrock surface in the Cleveland area provides information valuable to the civil engineer, and should be consulted by those interested in the area.(1)

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REFERENCE

1. Winslow, J. D., White, G. W., and Webber, E. E., The Water Resources of Cuyahoga County, Ohio, U. S. Geol. Survey, Water Resources Div. Bull. 26, Columbus, August 1953.

Discussion of
"EXPLORATION PRINCIPLES FOR
MAJOR ENGINEERING WORKS"

by W. R. Judd
(Proc. Paper 550)

W. R. JUDD,[1] A. M. ASCE—It appears that Mr. Fluhr has had some difficulty in obtaining appreciation of the benefit of geology in tunnel construction and design. Both with Government and with private engineers, the writer has found wide acceptance of the value of geology in designing tunnels, estimating their eventual cost, and predicting construction difficulties. Also, geological knowledge generally was welcomed when difficulties occurred during the operation and maintenance of the tunnel. Knowledge of whether the contractor is likely to hit water flows, and if so, in large or small quantities, would be of value in estimating the cost of the job; also it would enable the contractor to prepare for such troubles. Predictions of squeezing ground, excessive overbreak, roof falls, and invert heaves are valuable to the designer in preparing potential cost estimates, special design precautions, and evaluating possible delays in completion. Such delays could affect the ultimate use of other parts of the project which are dependent upon the tunnel. It always is possible that such tremendous difficulties would be encountered that abandonment of the tunnel might be necessary, or at the very least, excessive unforeseen cost. The geologist can utilize his experience to often solve these and other problems. Thus, it is difficult to understand why Mr. Fluhr seems to belittle the value of geology in tunnel construction. It is true that previously, unsuccessful attempts have been made to accurately correlate geology with tunnel-supporting practice; however, the present Task Committee of ASCE is the first coordinated effort toward the solution of that problem.

Very often, rigorous safety requirements do exert too much influence on the number of tunnel supports used. That is, although geological conditions may indicate the need for few or no supports, these conditions are ignored and supports are placed at regular intervals: A waste of steel that someone pays for. The emphasis by various authorities on support practice has resulted in research on both safety and economy. For example, the well-known roof-bolting practice has been a development of such emphasis.[2] Similarly, insurance companies and Government agencies have been responsible for successful research on microseismic methods of predicting the need for supports. By the latter method, it often is possible to predict when an

1. Engineering Geologist, Office of the Commissioner, Bureau of Reclamation, Denver, Colo.
2. Thomas, E., et.al., "Suspension Roof Support," USBM I.C. 7533, 1949; Thomas, E., "Roof Bolting in the U.S.," USBM I.C. 7583, 1950; Young, H. C., "Roof Bolting in Alabama Coal Mines and Iron-ore Mines," USBM I.C. 7678, 1954.

unsupported section will require the placement of supports to prevent roof falls.[3] Thus, efforts are being made to develop the placement of supports on a scientific and thus economical basis.

The writer does not deny that use of tunnel supports to some extent depends on factors extraneous to the geology. However, the writer firmly believes that these factors are in the minority when evaluating the necessity for supports, prior to construction. On tunnel jobs in certain isolated locations, it is necessary to arrive at such a decision before construction. For example, while assisting on the construction of a tunnel in Alaska,[4] the writer was requested to estimate the amount of support to be required. This estimate had to be primarily based on the geological situation; a minor percentage of steel was added to the estimate to account for the excess of supports usually required to satisfy the contractor's safety practice. In this case it was necessary, before construction, to know very closely, the total amount of steel that would be used, as the construction schedule had to provide for a 3-month shipping period. The use of geological studies thus prevented a costly excess of steel from being shipped and left over after the job was completed, or the ordering of an insufficient amount, and subsequent construction delays until the missing steel arrived.

It has been the writer's experience that geology has an important role in the location of tunnel lines, particularly if geological factors favor one route over another. Of course, such an analysis must also include the engineering and hydraulic factors involved. For example, several alternates were proposed for a tunnel through the Continental Divide in Colorado. A geological evaluation was made of each alternate; the results were analyzed on the basis of geologic, economic, and engineering factors. The final location was established where it would avoid a major fault zone that the geologist interpreted would occur at tunnel grade.

Another and more current use of geology is in the design of high-pressure water tunnels and underground penstocks. It is only in recent years in the United States that geotechnical knowledge has been utilized to evaluate stress conditions in the rock surrounding the tunnel. By taking advantage of these predicted stresses, the thickness of steel lining required or the amount of concrete reinforcement often can be reduced.

The writer believes that it is well established that certain types of rocks, particularly shales, and some siltstones and claystones, deteriorate upon exposure to air. While some deterioration undoubtedly is caused by "Massive" stress relief, the slaking usually seems to result from a change in moisture content. Most rocks contain some moisture in capillary, pellicular, or structural form. In the excavation when the rock is first exposed to air, atmospheric conditions may exert a dehydrating effect; thus, as the moisture leaves the rock, minute stresses develop in the rock fabric. These stresses often result in cracking and slaking particularly in softer sedimentary rocks. Similarly, if the rock is exposed to an atmosphere of high humidity, the air moisture may chemically attack or be sorbed by the rock. If the rock contains a high percentage of montmorillonite-type clay, for

3. Obert, L., and Duvall, W., "Microseismic Methods of Predicting Rock Failure in Underground Mining," Pt. I, USBM R.I. 3797, February 1945; Ibid, Pt. II, USBM R.I. 3803, March 1945.
4. Judd, W. R., "Foundation Problems of the Eklutna Project," ASCE Proc.–Sep. Paper 444, 1954.

example, the resultant increase in water content would cause expansion which could be evidenced by the slaking of the rock.

Mr. Fluhr's last statement that seems to deny the advantage of publications designed to assist an untrained engineering geologist is difficult to understand. He has undoubtedly found, as have all of us in the profession, that it is not always possible to obtain a thoroughly experienced engineering geologist to assist the engineer. Therefore the engineer is forced to rely on a less experienced man. If this geologist has no thoroughly experienced man to guide him, he must dive into the job completely ignorant of problems likely to arise. But, if he has some references at his disposal describing a range of engineering geologic problems and the experiences of others, he is better equipped to proceed. We all know that references alone do not "make" an engineering geologist, but they at least will help the "green" man. The geologist aware of his shortcomings can more safely operate than a geologist completely unaware of engineering geologic principles.

Discussion of
"EFFECT OF SAMPLE DISTURBANCE ON THE STRENGTH OF A CLAY"

by Max L. Calhoon
(Proc. Paper 570)

MAX L. CALHOON,[1] J. M. ASCE.—The writer is appreciative of Mr. Schmertmann's thoughtful discussion of the above subject and particularly of his fine explanation of Rutledge's hypothesis on the effects of sample disturbance on test results of undisturbed soil samples.

It may be interesting in connection with the phenomenon of the compressive strength increasing exponentially as the failure void ratio (or water content) decreases arithmetically, that in addition to saturated clays, it has been found that saturated peat soils behave in the same manner. Figure 1 shows a typical example. It can also be seen that the compressive strength curve is essentially parallel to the triaxial consolidation curve.

Mr. Schmertmann has presented some very interesting data on the strength of a remolded saturated clay which was remolded during construction operations. As can be seen in his Figure 3 the "a_r" value is negative. It may be seen in Figure 7 of the original paper that the "a" values for both the completely remolded and undisturbed specimens (approximately 50% remolded) are negative. It is the opinion of the writer that a negative "a" value is indicative of serious disturbance and when this occurs in presumably undisturbed samples, serious disturbance has actually occurred and correction for it should be made before the data is used in design.

Mr. Schmertmann makes the statement that he believes that Figure 8 of the original paper is in error and discusses the relative positions of the remolded and undisturbed shear strength envelopes. It is believed that Mr. Schertmann is confused as to the content of Figure 8 since the two curves shown are for laboratory undisturbed shear strength and the field shear strength. No remolded shear strength curve is shown.

The writer is in complete agreement with Mr. Schmertmann on the impracticability of the laboratory test series for the evaluation of strength data presented in the original paper. A simpler approach must be found. Mr. Schmertmann has proposed a tentative procedure but the writer does not favor it for several reasons. Schmertmann has gone to great length to extrapolate a field virgin consolidation curve which depends upon reliable knowledge of the natural void ratio and preconsolidation pressure in the field. It has long been known for normally loaded clays that if the natural void ratio and preconsolidation pressure in the field are known, that a virgin consolidation curve passing through this point and either parallel to or slightly steeper than the test laboratory virgin consolidation curve represents an acceptable field consolidation curve. It is not believed that Schmertmann's concept of the initial virgin slopes of all consolidation curves passing through a common

1. Soils Engr., Howard, Needles, Tammen and Bergendoff, New York, N. Y.

point at 42% of the initial void ratio is any more accurate or more acceptable than the above method. He has then further assumed that all void ratio versus log compressive strength curves pass through a common point and gives such a point for the data in the original paper as 33% water content and 9 tons/sq. ft. compressive strength (See Figure 7 of original paper). If the field strength curve were not already plotted on the same graph (Figure 7) for reference, it is believed that a different point, say 34% and 4 tons/sq. ft. would logically have been chosen and would result in a field strength curve that would be quite different from that shown in Figure 7.

Mr. Schmertmann also stated in his method that unconfined compression samples could be cut large enough so that trimming disturbance would be approximately the same (equal surface areas/unit volume) as that of the least disturbed consolidation test sample. To do this, Schmertmann is apparently assuming that the depth of remolding in a triaxial specimen is approximately the same as that in a consolidation specimen due to the sampling and trimming operations. In referring to Figure 1 it will be seen that the triaxial consolidation curve is below the one-dimensional consolidation curve. The dimensions of the triaxial specimen were 1.4 inch diameter by 2.8 inches in length and the dimensions of the one-dimensional consolidation specimen were 2.5 inches in diameter by 0.75 inch in thickness. Therefore, if the remolding was to be the same depth in both specimens, the triaxial specimen should have been remolded less than the one-dimensional and the triaxial virgin curve should have been above the one-dimensional virgin curve. It is concluded that the two different trimming operations do not produce comparable disturbance and therefore a consolidation curve representing the same amount of remolding as the strength curve represents is a necessity to the strength evaluation procedure so that the amount of remolding can be calculated in the strength specimens.

The writer would now like to take the liberty of proposing an untried strength evaluation procedure in the interest of sharing his ideas on the subject with other engineers with the hope that eventually a simple and reliable strength evaluation procedure will be found. While this method has never been tried in the laboratory, all available evidence strongly indicates that it will prove workable. This proposed method requires two one-dimensional consolidation tests, two triaxial shear tests and as many unconfined strength tests as necessary to define two strength curves. Four unconfined compression tests would probably be a minimum feasible for use.

It was shown by Van Zelst (reference 11 of original paper) that all significant remolding to one-dimensional consolidation samples occurs in trimming and planing the horizontal surfaces of the specimens. This is revealed by the fact that specimens of various thickness show virgin consolidation curves that are displaced vertically such that equation 1 and 2 (see below) are satisfied. Therefore, peripheral disturbance is apparently negligible. In view of the foregoing, a simpler method of extrapolating the field virgin consolidation curve than that proposed by Van Zelst and used in the original paper has been found.

If it is assumed that the depth of remolding in one-dimensional consolidation specimens of the same area is equal in specimens of all thicknesses, it can be shown that the ratio of percent disturbance in a thin specimen to that in a thick specimen is proportional to the ratio of the thickness of the thick specimen to that of the thin specimen. In equation form:

$$R = \frac{T1}{T2} \qquad (1)$$

where: R = Ratio of disturbance of thin specimen to thick specimen

T_1 = Thickness of thick specimen

T_2 = Thickness of thin specimen

In referring to Figure 2 it may be seen that the vertical distance from curve 1 to curve 3 must be R times the vertical distance from curve 1 to curve 2 since distances in the vertical direction are proportional to percent disturbance. In equation form:

$$R = \frac{B + X}{X}$$

or

$$X = \frac{B}{R-1} \qquad (2)$$

(see Figure 2 for terminology)

Since R and B are known quantities, X can be computed at various pressures and the location of the field virgin consolidation curve can be plotted.

To evaluate the strength characteristics two strength curves are required, one obtained from small compression specimens, say 1.4 inch diameter by 2.8 inch length and the other from large compression specimens, say 2.8 inches in diameter by 5.6 inches in length. At least one small specimen and one large specimen must be tested in triaxial shear to obtain triaxial consolidation curves. The remainder of the strength specimens may be consolidated by slow drying in a humid room and tested by unconfined compression as suggested by Schmertmann. With the positions of the field consolidation curve and the two triaxial consolidation curves known, the R value for triaxial consolidation can be computed. The triaxial R value is then used to compute the position of a field strength curve from the two known strength curves by use of equation 2 in a similar manner as the position of the field consolidation curve was computed.

In summary, the procedure proposed herein consists of the following:

(1) Perform one consolidation test in a thick ring, one consolidation test in a thin ring, one triaxial test on a large specimen, one triaxial test on a small specimen, at least two unconfined compression tests on large specimens and at least two on small specimens (it is to be emphasized that the unconfined compression specimens must be trimmed by the same procedure as the triaxial specimens were trimmed). The foregoing is a minimum of testing and more may be required depending on the uniformity of the soil being tested.

(2) Plot all test results on the same graph.

(3) Plot the position of the field consolidation curve by use of equations 1 and 2.

(4) Compute the R value for triaxial consolidation by use of the field consolidation curve, the two triaxial consolidation curves and equation 2.

(5) Plot the field strength curve by use of the triaxial R value found in step (4) and equation 2 where the X and B values are referred to the two laboratory strength curves.

It is to be emphasized that this procedure has never been tried in the laboratory but is proposed in order to provoke more thought on the subject by all interested persons.

ACKNOWLEDGMENTS

The data presented in Figure 1 was obtained from the files of the Soils Laboratory of Howard, Needles, Tammen and Bergendoff, New York, N. Y.

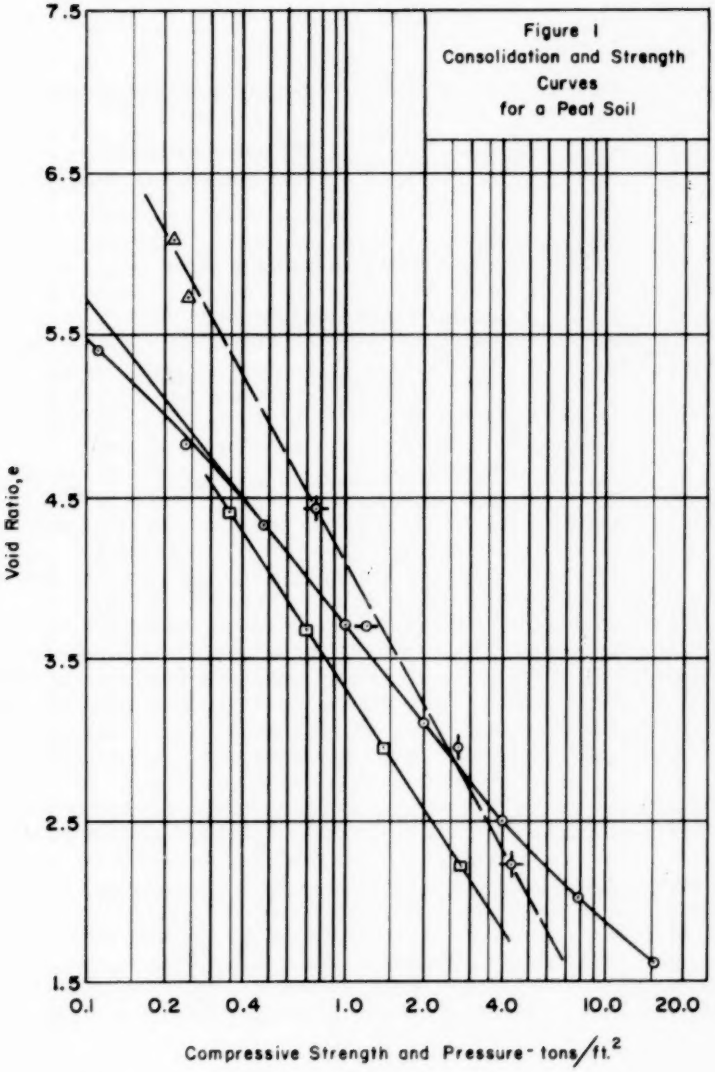
Figure 1
Consolidation and Strength
Curves
for a Peat Soil
7.5
6.5
5.5
4.5
3.5
2.5
1.5
Void Ratio, e
0.1
0.2
0.4
1.0
2.0
4.0
10.0
20.0
Compressive Strength and Pressure - tons/ft.²
Notes:
o One-Dimensional Consolidation
□ Triaxial Consolidation
——Compressive Strength

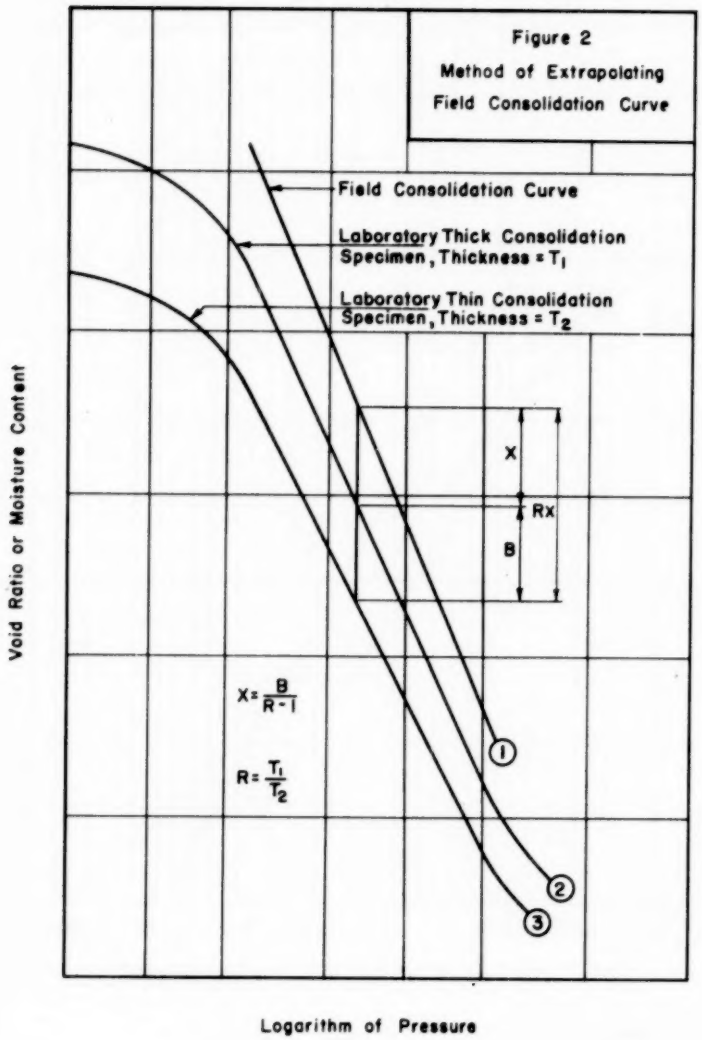
Figure 2
Method of Extrapolating
Field Consolidation Curve
Field Consolidation Curve
Laboratory Thick Consolidation
Specimen, Thickness = T_1
Laboratory Thin Consolidation
Specimen, Thickness = T_2
X
Rx
B
$X = \frac{B}{R-1}$
$R = \frac{T_1}{T_2}$
1
2
3
Void Ratio or Moisture Content
Logarithm of Pressure

Discussion of
"ROCK WEATHERING CLASSIFICATION OF EXCAVATION SLOPES"

by J. D. Welch
(Proc. Paper 754)

DART WANTLAND.[1]—The problem of the author (page 754-1) was similar in certain respects to the problem of measuring the depth of weathered rock on which the writer used geophysical methods. The electrical resistivity method and the seismic refraction method were employed to determine the thickness of weathered and disintegrated rock (that would have to be stripped before building a dam) at a site on the American River near the town of Folsom, California.

The results of field tests on this problem are described in the writer's paper "Geophysical Measurements of the Depth of Weathered Mantle Rock," A.S.T.M. Special Technical Publication No. 122, 1952.

The position of the "sound rock line" at the dam site studied was determined directly by core drilling. The cores were very carefully analyzed, by an engineering geologist, to determine specifically the degree of weathering of the rock and the depth to sound rock.

Resistivity (depth) measurements identified a high resistivity layer at very nearly the same depth as the sound rock. Seismic field measurements identified a layer which carried seismic waves at a high velocity which was likewise at essentially the same depth as the sound rock. Geophysically determined sound rock was at an average depth of 42.4 feet for the nine check points available. The average drill depth to sound rock, at these points, was 39.4 feet. The difference between these figures, of only 3 feet, was less than 8 percent of the drill depth.

The rock of the area was a granite or granodiorite and a slaty dark gray, somewhat contorted, schist; so that the results of the geophysical measurements apply to those two rock types.

The writer showed that rock weathering was related to both resistivity and seismic wave velocity. To the extent that weathering generally increases the clay content of rocks and formations it would tend to lower their electrical resistivity. It is also known that weathered rock shows a lower speed of travel for seismic waves than fresh rock. These relationships might be an index of the degree of weathering and thus useful in Mr. Welch's problem.

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Discussion of
"FIELD VANE SHEAR TESTS OF SENSITIVE COHESIVE SOILS"

by Hamilton Gray
(Proc. Paper 755)

E. VEY,[1] A. M. ASCE.—The tests described in this paper represent a valuable addition to our information on the usefulness and reliability of the vane apparatus for determining the true shear strength of clay soils. The author is to be complimented for the painstaking care he exercised in preparing samples for laboratory tests and in performing the field tests.

In reference to sample disturbance created by the insertion of thin-walled sampling tubes into the soil, it should be pointed out that a similar disturbance is produced by the insertion of the vane. In a series of laboratory tests which the writer and his associates performed on remolded soils the friction between the vane blades and the soil was evidenced by a distinct depression produced in the soil surface by the insertion of the vane. This depression occurred in the three clays tested and at all consistencies. It was greatest however when the moisture content of the clay was such as to cause the greatest stickiness. It was believed that this type of disturbance created stresses in the soil which were present when the vane shear test was made. At all events it seemed reasonable that the disturbance was likely to have some influence on the magnitude of the vane shear. To check this, two vanes of different lengths but of the same diameter were used in the same soil and the results compared. The two vanes are shown in fig. 1 and the results in table 1. In the large vane such as the author used in his field tests the cylindrical surface on which shear occurred is farther from the central portion of the vane than in the small vane. Since the disturbance due to insertion is greatest near the center of the vane the large vane values are probably less affected by this factor. It is believed that the thickness of the vane blades and the size of the vane shaft are not as important as the diameter of the vane itself.

The actual shear resistance as measured by the vane then, depends on a number of elements

1. The diameter of the vane.
2. The length of the vane.
3. The stickiness of the soil.
4. The shear strength of the soil.

It is believed that vane shear values cannot be accepted as true unless all of these factors are properly evaluated.

The author states that "It is but logical to believe that the difference observed between the results of field and laboratory vane tests are caused solely by sampling operations and that the difference between the laboratory vane

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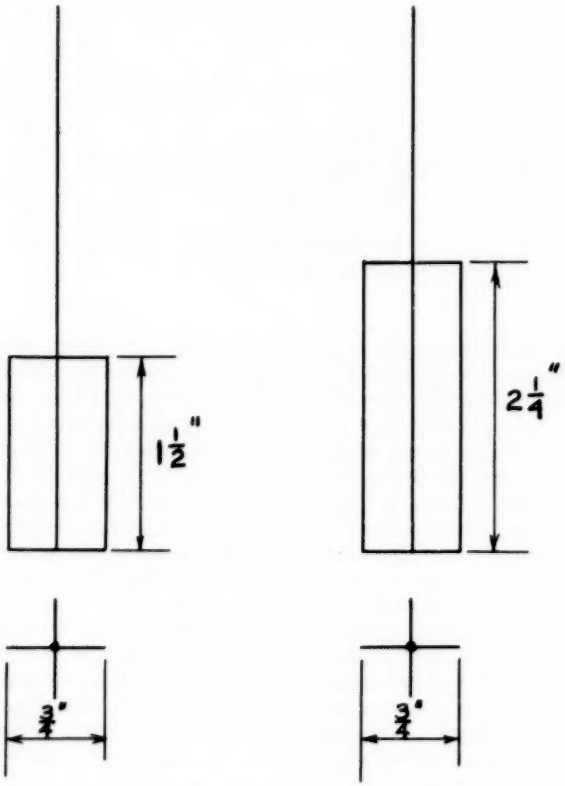

FIG. 1.

TABLE I

TEST NO.	TYPE OF VANE	WATER CONTENT %	MOMENT IN.#	$M_{11}-M_1$ IN.#	SHEAR #/IN.^{2}
1	I	25.7	1.685		1.17
	II	25.7	2.260		1.07
				0.575	0.86
2	I	20.1	10.50		7.30
	II	20.6	13.86		6.55
				3.36	5.00

and unconfined compression results are largely attributable to the preparation of the compression specimens." Because of the factors named above the same size and shape vanes would have to be used in the laboratory on samples of the same soil tested in the field to give a valid comparison.

There is an additional factor in this case which would tend to give the lower values which the author obtained using the small vane in the laboratory. The deposit tested was a varved clay consisting of alternate thin layers of clay, silt and fine sand. The vane shear in the field measured not only the shear strength of the clay but also the shear strength of the silt and sand. This would be a function of depth because of the increasing intergranular pressure with depth produced by the overburden. And because of the confined state of the laboratory sample the cohesionless material would again be responsible for some, although much less, of the vane shear resistance. In the case of the unconfined compression test there is no confinement of the cohesionless layers and the shear strength by this method would be almost entirely that of the clay and would, therefore, be expected to be much less than both the laboratory and the field vane strengths. The unconfined compressive strength moreover, would be expected to remain fairly constant with depth, within the depth where the pre-consolidation pressure was about the same. A more realistic measure of the strength for comparison with the vane data in this type of soil would be the tri-axial shear. The data for boring 4 shown in fig. 5 and 6 indicate that the increase in field vane shear over both the laboratory vane shear and the unconfined compression shear are more pronounced at the lower depths. It is also noted that the plasticity index values decrease with depth which would indicate an increasing proportion of silt and sand. This would reduce the unconfined compressive strength even further while the field vane shear would continue to increase with overburden pressure and the laboratory vane shear would show a smaller increase. There is considerable doubt that the vane results are reliable in highly silty and sandy soils.

It is believed that the consolidation samples prepared and tested by the author did give reliable pre-consolidation loads and that the disturbance of sampling and trimming consolidation samples were really only of secondary importance. The fact that as the author states "laboratory vane test results approximate, for example, but 3/4 of the corresponding field vane shear . . ." is to be expected in this type of soil and, "When the corresponding unconfined compressive strengths approximate only 50 per cent of the field vane strengths . . ." it does not necessarily mean that the sample were dangerously disturbed and consolidation data unreliable.

The author states that at the 80 ft. depth the unconfined compressive strength was over 0.4 tons/sq. ft. compared with less than 0.3 tons/sq. ft. throughout the rest of the depth and that in this region the laboratory vane results rather closely approximate the field vane results. He suggests that this soil was not as sensitive or may not have been damaged so much in sampling. The boring log description near this depth fig. 5 shows "many black organic stains." This would indicate that this soil might have had a higher organic content than the soil above and below. This would provide a binder for the cohesionless material and thus give to the soil mass a high cohesive strength after having been consolidated under the consolidation pressure of 2 tons/ft.^{2}. This mixture would accordingly derive its shear strength mainly from this cohesion and under the circumstances all three shear values would be expected to be closer together.

The writer believes that the author has over emphasized the effect of sample disturbance in evaluating the discrepancies between the shear strengths obtained by the three methods. It is believed, further, that the differences in strength as measured by the three methods were due principally to the varved nature of the clay rather than the disturbance of sampling. The nature of the vane disturbance may have also been a factor in comparing the results obtained by the small vane on laboratory samples with those of the large vane in the field.

CARL W. FENSKE,[2] A. M. ASCE.—Prof. Gray is commended for this paper, the stated purpose of which is to stimulate interest in the use of the field vane to secure in-place shear strengths of soils. This device has its definite place in the equipment that a soils engineer may use to obtain adequate and economical measurement of the shearing strength of soils. The purpose of this discussion is to present additional information on the comparison of shear strengths as measured by the vane, both miniature and field, and by more conventional laboratory tests.

The writer has available shear strength data where comparisons can be made between the results of miniature vane and unconfined compression tests. This information was obtained at a number of locations in the Gulf of Mexico offshore from the coast of Louisiana. The soils are post-glacial deposits of the Mississippi River and, therefore, are of Recent Age. They are less sensitive than the soils studied by the author with the degree of sensitivity averaging about 2.5. The miniature vane tests were made by inserting a four-blade vane, 0.760 inches in diameter and 0.685 inches long, into the end of the Shelby tube sample as soon as it was recovered from the boring. The vane was slowly rotated and the maximum torque required was observed. Rotation was continued and the minimum torque required to produce a continuous rotation was also observed. Maximum and minimum vane strengths were computed from these observed torque values. Unconfined compression tests were made at a constant rate of strain of about 2% per minute. A good agreement was found between the maximum miniature vane strengths and the shear strengths from undisturbed unconfined compression tests for soils firm enough to be extruded from the sampling tube and prepared for testing without excessive disturbance. The only exceptions to this agreement have been when the tests were made on soils so soft that extrusion and preparation for the unconfined compression test would not be performed without noticeable disturbance. A similar good agreement has been found also between the minimum miniature vane strength and the shear strength from remolded unconfined compression tests.

Investigations with a field vane 3 inches in diameter and 6 inches high were made at a few locations along with miniature vane tests and tests on undisturbed samples. In these investigations, the agreement between the results of miniature vane and undisturbed unconfined compression tests mentioned above was obtained but significantly higher shear strengths were obtained by the field vane. However, the results of consolidated-undrained triaxial tests performed on undisturbed samples, with a lateral pressure equivalent to the computed effective overburden pressure at the depth from which the sample was obtained, were comparable with the field vane test results.

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In the light of the above information, the writer agrees with Prof. Gray's conclusion that the difference between the undisturbed unconfined compression and laboratory vane results for the sensitive soil studied is primarily caused by unavoidable disturbances in sample preparation. However, it is suggested that structural disturbance during sampling may not be the only factor causing the difference between the field vane and the laboratory vane strengths. There may be a more fundamental factor such as, possibly, the stress-relief to which the sample is subjected in bringing it to the surface. This would mean, first, that even if tests were made on undisturbed samples that were perfect with respect to structural disturbance, there would still be an intrinsic difference between the results of field vane and unconfined compression tests; and, second, that an agreement, especially at appreciable depths, between the results of these two tests should not be expected.

DALE T. HARROUN,[3] A. M. ASCE.—Any technique of performing soil tests in the field is highly desirable not only from the standpoint of minimizing sample disturbance, but also in the saving of time and expense.

Field soil tests, however, are subject to all the stresses and effects existing in a natural soil stratum, without any screening or idealizing of conditions such as may occur, inadvertently or otherwise, in a laboratory.

A vane shear test is basically a direct torsion shear test performed in a horizontal plane with a vertical axis of twist. In any direct shear test on typical soils having both cohesion and friction:

$$\text{Shear} = C + N \tan \phi$$

In the case of the vane shear, the normal stress "N" is equal to the lateral soil pressure, which is in turn a function of the vertical soil pressure at that point. For a purely cohesive soil it also follows that the value of shear would be independent of the lateral soil pressure. However, a purely cohesive soil is a somewhat academic condition, most soils having at least a small angle of friction. The author mentions the silt content of the subject soil, for example.

Referring to the author's test data in Figure 6, it is shown that the value of laboratory unconfined compression tests on the soil samples, free of any enclosing lateral stresses, displays very little increase of strength with depth.

On the other hand, field vane shear results show an appreciable increase of strength in direct proportion to increase in lateral pressure.

The laboratory vane shear tests conducted in the sample tubes display this direct increase of strength with depth to a lesser extent, due perhaps to the retention of some of the natural lateral soil pressure by the sample tube.

Since the vane shear, like the unconfined compression test, measures only total shear, it is not possible to differentiate between the frictional and cohesive components. If conventional direct shear tests results were available, a more analytical study of the problem could be made.

The increase of field shear with lateral stress due to depth could also explain the author's observation under "Significance of Results" where the increase of shear value is attributed to a possible natural increase of soil strength with depth.

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Discussion of
"SETTLEMENT ANALYSIS OF SAND
DRAIN PROJECTS"

by Edward A. Henderson
(Proc. Paper 756)

EARL M. BUCKINGHAM[1] A. M. ASCE AND FRANK G. CHENNEY[2].— The author has presented a careful study of measured fill settlements in the light of current theories and has noted some important discrepancies between these theories and the behavior of constructed fills. It is only by such studies that the profession is able to evaluate its methods and lay the groundwork for further advances. The theory of soil consolidation has been developed from a mathematical approach necessarily based on certain assumptions and approximations, the errors of which have not yet been fully evaluated. The fact that a method of analysis gives results in agreement with the behavior of constructed works is not in itself proof of the validity of the method. One has only to consider the number of widely different methods which have been proposed for predicting the behavior of piles to realize that a method having serious shortcomings may give surprisingly correct results under favorable conditions. The same is true of several different conceptions of the forces and resistances involved in landslides.

In order to obtain workable equations the theories of consolidation make several assumptions which may be at considerable variance with prototype conditions. Considering first the one dimensional case of simple vertical drainage, it has been necessary to assume a constant permeability. Obviously as soon as consolidation begins porosity decreases and permeability will no longer be constant with respect to either time or depth. This factor would affect both prototype and sample, but might cause considerable divergence from the theoretical curve for some soils.

The author suggests that the so-called secondary consolidation may at times be of considerable importance. This appears to be particularly true for certain soft muds which may undergo one or more cycles of collapse of intergranular structure. This action is accelerated by vibration or shock loading such as occurs when traffic leaves a culvert or other structure supported by piles.

Passing to the three dimensional problem of sand drains the divergence between prototype and mathematical assumptions becomes more pronounced. The accepted conception, on which the author has based his study, is one of a homogeneous but not isotropic material which acts as a single unit throughout its entire depth. Most sedimentary deposits are stratified as a result of both annual and secular cycles. In marine or bay deposits annual variations alone tend to result in a uniformly varved material which should behave in agreement with current theory and deposits of this type should continue to be

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analyzed as has been done in the author's paper. However most deposits contain irregularly spaced sand lenses, shell beds, or peat seams, all having considerably greater permeability than the body of the deposit. It is the writers' belief that the principal function of sand drains is the relief of pressure built up in these lenses, thence the problem of predicting time of settlement in deposits of this latter type should be approached by determining the number, spacing, areal extent, and at least estimated permeability of the lenses and computing the one dimensional consolidation of the mud between them. However no forseeable method of prior analysis can destroy the validity of the author's conclusion No. 3 regarding the value of field observations.

If the purpose of sand drains is to relieve pressure in relatively permeable lenses the diameter of the drain ceases to be an important consideration so long as clogging or other damage does not occur. This suggests the use of closely spaced inexpensive drains such as wash borings made after the sand blanket has been placed. Other factors in favor of wash borings are the absence of heaving of completed drains and the elimination of distortion or destruction of thin drainage seams by driving operations.

In conclusion the writers wish to make clear that it is not their intent to criticize present methods or advocate their abandonment, but to point out that further refinements are possible as more field observations become available.

CHARLES M. NOBLE,[3] M. ASCE.—The Route 35 (Point Pleasant) Project had interesting and challenging possibilities. The failure by teredo action of the low level drawbridge on the Causeway connecting Point Pleasant and Brielle, New Jersey, necessitated prompt action in building a new line to provide for extremely heavy land and water traffic. There was no other crossing within some miles of the site. Replacement was the more urgent since the failed bridge could be temporarily repaired for emergency use only in the closed position, thus inconveniencing a large and active pleasure and fishing fleet. Because of heavy vehicular traffic and consequent serious tie-ups caused by frequent raising of the drawbridge, it was determined that the new facility must be constructed at a higher level, so as to pass more than 90% of the waterborne traffic without requiring the draw to be lifted.

The failed bridge and the channel lay along the Brielle shore where foundation conditions were sandy and sufficiently stable that special treatment was unnecessary. From the south end of the old draw over the channel, a low earth fill causeway extended 1300 feet toward Point Pleasant, across shallow water with a soft mud bottom, some 40 feet deep, described by the Author. The causeway, constructed 25 years before, was "floating" on this mud.

At first, it was thought the only solution was to build a structure almost entirely across the waterway due to the required 35 foot clearance of the new drawbridge over the channel and the fact that the existing causeway and repaired span must be kept in service continuously until the new facility was completed and opened to traffic. It was not feasible to move the new roadway sufficiently distant to avoid complications with the "floating" causeway and permit normal fill operations. The 40 feet of mud was soft, with a shear strength of 200 to 400 pounds per square foot and a water content of 60 to 150% of the dry weight, which indicated that it would not be feasible to construct a 35 to 45 foot fill by ordinary methods without the danger of major displacement and slides. Such failures would have disrupted the existing

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Causeway and also may have caused serious movement towards the channel, possibly further damaging the old bridge and the proposed abutment and piers for the new bridge. Obviously it was not feasible to dredge out the mud to a depth of 40 feet and refill the excavation with stable sand by methods successfully used to moderate depths on other projects because of the likelihood of the old Causeway sliding into the trench.

The spanning of the entire waterway with bridge structure was so costly, however, that additional studies were made and upon the recommendation of O. J. Porter, Consulting Engineer, a serious study was made of the sand drain method of mud stabilization as an alternate to the structure. This led to the adoption of the sand drain stabilized fill project described by the Author.

The problem was not only to stabilize rapidly the soft mud foundation under the heavy fill to preclude excessive settlement after the pavement was placed but of equal importance, to do so without lateral and vertical movement of the immediately adjoining Causeway, which was carrying continuously a heavy flow of traffic. For complete success, it was also essential that the fill and its foundation be stabilized to a high degree so that there would not be any movement towards the new bridge, either during or subsequent to construction. To accomplish this, the vertical sand drains were installed on 7 foot centers in the area immediately below and adjacent to the proposed new bridge abutment, at 8 foot centers below the high portion of the embankment, and at 10 foot centers under the slopes and lower portions of the embankment where the fill pressures and required shear strengths were less. Figure 1 of the Paper shows the critical relationship between the new fill and the Causeway.

This sand drain project was carried out successfully as described by the Author at an estimated saving of $1,000,000. compared with a structure spanning the entire waterway.

At Camden, the problem also involved a comparison with a structure solution, for not only was the problem of foundation consolidation important but it was imperative to assure there would be no movement or slide into the immediately adjoining navigable channel of the Cooper River. The project lay within a narrow strip between the River and Admiral Wilson Boulevard (U. S. 30), which carries 85,000 AADT vehicles per day. Here again the work was carried out successfully at a considerable cost saving.

The Author is to be commended for presenting this Paper so the profession will have the benefit of the experience derived from these two difficult and exacting projects.

WILLIAM G. WEBER, JR.,[4] A.M. ASCE.—The author has presented an excellent theoretical solution to rate of settlement when sand drains are used. This method, proposed by Reginald A. Barron in 1948, has been in use by the California Division of Highways for several years. The estimation of the actual rate of consolidation by means of the pressure ratio has been used on several recent projects with excellent results. As all of this work is done during the design of the project the horizontal permeabilities have to be measured. Using the above data, the amount of overload and time it should be left in place is determined. These estimates are then checked in the field when the projects are constructed against the observed settlements.

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It must be realized that the prediction of settlements in sand drain areas is subject to all of the limitations and assumptions of the theory of consolidation. Also that small volumes of soil are tested and are assumed to represent great volumes of soils in place, and that soils are very seldom homogeneous, which may affect the results. Thus, by their very nature, any settlement calculations are subject to considerable error. In the numerical example given on page 756-10, taking Cv as the extreme high value of 0.07 ft.^{2} per day instead of the average value of 0.05 ft.^{2} per day an estimated settlement of 5.0 feet is obtained at 650 days, instead of 4.5 ft. by using the average value of Cv, with the time settlement curve following the Cv = 2Cv curve of Fig. No. 10. It is thus apparent that the accurate determination of Cv has a small influence upon the rate of settlement. As Cv and Cr are dependent upon the vertical and horizontal permeabilities it appears that the determination of the magnitude of the permeability ratio is of the greatest importance. Knowing the ratio of horizontal to vertical permeabilities will enable the designing engineer to determine the required sand drain spacing, required height of surcharge, and estimated time that the surcharge is to remain in place. Thus, an estimate of the relative costs and/or effectiveness of sand drains can be determined.

In the author's conclusion No. 3 it is true that the correct values of Cv and Cr are difficult to obtain. However, the statement, leaving the decision as to the amount and time for the overload to remain in place until actual construction, places sand drains in the class of an experimental installation. After all the years that sand drains have been in use, it should be possible to estimate in advance if they will be effective. Where the contractor has a time limit upon completing his project, this could result in considerable difficulties. To the writer it would appear that some other method of stabilizing soft soils is desirable if it will be necessary to make each sand drain installation a "test section." A reasonable approach to this problem would be the development of test methods and procedures to measure the necessary soil properties.

Conclusion No. 4 is based entirely upon theoretical considerations. It would be interesting to know if settlement platforms were placed in similar fills without sand drains, and what these settlement platforms indicate. It has been the writer's priviledge to review several sand drain projects, and on several projects where settlement records were available in non-sand drain areas there does not appear to be an appreciable difference in the rate of settlement after construction. It would appear that a reasonable measure of the success of sand drains, in relation to settlement, can only be told from a comparison of settlement records in sand drain areas and in non-sand drain areas. The literature contains many articles with statements of this nature, based entirely upon theoretical consideration, but with no factual data to substantiate them.

In conclusion the writer feels that a greater effort should be devoted to developing methods and procedures of testing the soil so that the designing engineer can properly evaluate how sand drains may be expected to operate. The general theories of design have been available for several years and there is now a need for more detailed descriptions giving numerical comparisons of settlement rates as predicted and as observed in construction.